

## THE NWS SIMPLIFIED DAM BREAK FLOOD FORECASTING MODEL FOR DESK-TOP AND HAND-HELD MICROCOMPUTERS

Jonathan N. Wetmore and Danny L. Fread<sup>1</sup>

### SYNOPSIS

The National Weather Service (NWS) has developed a simplified procedure that utilizes desk-top or hand-held microcomputers for predicting downstream flooding produced by a dam failure. This procedure, known as the Simplified Dam Break (SMPDBK) Flood Forecasting Model, produces information needed for delineating areas endangered by dam break floodwaters while substantially reducing the amount of time, data, computer facilities, and technical expertise required in employing more highly sophisticated unsteady flow routing models. With only an inexpensive microcomputer and a minimal amount of data, the user may within minutes predict the dam break floodwave peak flows, depths, and travel times at selected downstream points. This capacity for providing results quickly and efficiently should make the SMPDBK model a useful forecasting tool in a dam failure emergency when warning response time is short, little data are available, and large computer facilities are inaccessible. However, the SMPDBK model should prove even more useful for "pre-event" dam failure analysis by emergency management personnel engaged in preparing disaster contingency plans when the use of other flood routing models is precluded by limited resources.

The SMPDBK model is designed for interactive use (i.e., the computer prompts the user for information on the dam, reservoir, and downstream channel and the user responds by entering the appropriate data values), and it allows the user to enter as much or as little data as are available, automatically using preprogrammed defaults when the response to a prompt indicates the data is not available. Using the internally set default values, SMPDBK is capable of producing approximate flood forecasts after reading in only the dam height, reservoir storage volume, and depth-vs.-width data for one cross-section of the downstream river valley (determined from on-site inspection or topographic maps). If however, the user has access to additional information (i.e., the reservoir surface area, estimates of the final width and depth of the breach, the time required for breach formation, the turbine/spillway/overtopping flow, the slope of the channel and the Manning roughness coefficient, flood depth (depth where flooding becomes a problem), and elevation-vs.-width data for up to five downstream channel cross-sections), the model will utilize this information to enhance the accuracy of the forecast.

---

<sup>1</sup>Research Hydrologist and Senior Research Hydrologist, Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA, 8060 13th Street, Silver Spring, Maryland.

In producing the dam break flood forecast, the SMPDBK model first computes the peak outflow at the dam, based on the reservoir size and the temporal and geometrical description of the breach. The computed floodwave and channel properties are used in conjunction with routing curves to determine how the peak flow will be diminished as it moves downstream. Based on this predicted floodwave reduction, the model computes the peak flows at specified downstream points with an average error of +10 percent or less. The model then computes the depth reached by the peak flow based on the channel geometry, slope, and roughness at these downstream points. The model also computes the time required for the peak to reach each forecast point and, if the user entered a flood depth for the point, the time at which that depth is reached as well as when the floodwave recedes below that depth, thus providing the user with a time frame for evacuation and fortification on which the preparedness plan may be based.

The SMPDBK model has performed well in test simulations of the flooding produced by the failure of Teton Dam and the Buffalo Creek "coal waste" Dam, as well as in numerous theoretical dam failure simulations where the progression of the floodwave was not significantly altered by backwater effects created by downstream dams or bridge embankments, the presence of which can substantially reduce the model's accuracy. Its speed and ease of use recommend it well for use in emergencies. However, emergencies are not the only situations where it can be useful; planners, designers, emergency managers, and consulting engineers responsible for predicting the potential effects of a dam failure may employ the model in situations where backwater effects are not significant for pre-event delineation of areas facing danger should a particular dam fail.

## I. INTRODUCTION

The devastation that occurs as impounded reservoir water escapes through the breach of a failed dam and rushes downstream is quick and deadly. This potential for disastrous flash flooding poses a grave threat to many communities located downstream of dams. Indeed, a report by the U.S. Army (1975) indicates 20,000 dams in the U.S. are "so located that failure of the dam could result in loss of human life and appreciable property damage ...". This report, as well as the tragic destruction resulting from the failures of the Buffalo Creek coal-waste dam, the Toccoa Dam, the Teton Dam, and the Laurel Run Dam, underscores the real need for accurate and prompt forecasting of dam-break flooding.

Advising the public of downstream flooding during a dam failure emergency is the responsibility of the National Weather Service (NWS). To aid NWS flash flood hydrologists in forecasting the inundation resulting from dam-failures, the numerical NWS Dam-Break Flood Forecasting Model (DAMBRK) (Fread 1977, 1980) was developed for use with large, high-speed computers to model the outflow hydrograph produced by a time-dependent, partial dam breach, and route this hydrograph downstream using the complete one-dimensional unsteady flow equations while accounting for the effects of downstream dams, bridges, and off-channel storage. However, in some situations the real-time use of the DAMBRK model may be precluded because warning-response time is extremely short or adequate computing facilities are not available.

To alleviate this potential problem and attempt to improve upon the accuracy and versatility of existing simplified dam breach modelling procedures, the NWS has developed the Simplified Dam-Break (SMPDBK) Flood Forecasting Model. With this model, the user may within minutes produce forecasts of the dam-break floodwave peak discharges, stages, and travel times. It should be noted, however, that the use of the NWS SMPDBK model is not limited to NWS flash flood hydrologists. Planners, designers, civil defense officials, and consulting engineers who are concerned with the potential effects of a dam failure and who have limited time, resources, data, computer facilities, and/or experience with unsteady flow models may also wish to employ the model to delineate the areas facing danger in a dam-break emergency.

This document presents an outline of the NWS SMPDBK model's conceptual basis. Appendix I gives a step-by-step guide and example of the computations involved in the model. Appendix II presents the FORTRAN computer code for the automated (mini/micro-computer) version of the model and an example run of that version. Appendix III presents the BASIC computer code for the microcomputer or programmable hand-held computer version of the model. Appendix IV presents the HP41C hand-held computer version of the model.

## II. MODEL DEVELOPMENT

The SMPDBK model retains the critical deterministic components of the numerical DAMBRK model while eliminating the need for large computer facilities. SMPDBK accomplishes this by approximating the downstream channel as a prism, neglecting the effects of off-channel storage, concerning itself with only the peak flows, stage, and travel times, neglecting the effects of backwater from downstream bridges and dams, and utilizing dimensionless peak-flow routing graphs developed using the NWS DAMBRK model. The applicability of the SMPDBK model is further enhanced by its minimal data requirements; the peak flow at the dam may be calculated with only four readily accessible data values and the downstream channel may be defined by a single "average" cross-section, although prediction accuracy increases with the number of cross-sections specified.

Three steps make up the procedure used in the SMPDBK model. These are: (1) calculation of the peak outflow at the dam using the temporal and geometrical description of the breach and the reservoir volume; (2) approximation of the channel downstream of the dam as a prismatic channel; and (3) calculation of dimensionless routing parameters used with dimensionless routing curves to determine the peak flow at specified cross sections downstream of the dam.

### 2.1 Breach Description

Most investigators of dam-break flood waves have assumed that the breach or opening formed in a failing dam encompassed the entire dam and occurred instantaneously. While this assumption may be valid for a few concrete arch dams, it is not valid for the exceedingly large number of earth dams. Because earthen dams generally do not fail completely nor instantaneously, the SMPDBK model allows for the investigation of partial failures occurring over a finite interval of time. And, although the model assumes a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of

the trapezoidal breach. Failures due to overtopping of the dam and/or failures in which the breach bottom does not erode to the bottom of the reservoir may also be analyzed by specifying an appropriate "H" parameter which is the elevation of the reservoir water surface elevation when breach formation commences minus the final breach bottom elevation (i.e., "H" is the depth to which the breach cuts).

The model uses a single equation to determine the maximum breach outflow and the user is required to supply the values of four variables for this equation. These variables are: 1) the surface area of the reservoir; 2) the depth to which the breach cuts; 3) the time required for breach formation; and 4) the final width of the breach. (Note: For "pre-event" analyses, the user must estimate the last three variables above. To assist in this estimation, the following table of default values is provided.)

Table 1--Default Values for Breach Description

<u>Value</u>	<u>Units</u>	<u>Description</u>	<u>Default</u>
H	ft	Depth to which breach cuts	Height of dam
B <sub>f</sub>	ft	Final breach width	3 x breach depth (earth dams) 1/4 - 1/2 dam width (concrete gravity dams) Entire dam width (concrete arch dams)
t <sub>f</sub>	minutes	Time of failure	H/3 (earth dams) H/1000 (concrete dams)

Once the maximum outflow at the dam has been computed, the depth of flow produced by this discharge may be determined based on the geometry of the channel immediately downstream of the dam, the Manning "n" (roughness coefficient) of the channel and the slope of the downstream channel. This depth is then compared to the depth of water in the reservoir to find whether it is necessary to include a submergence correction factor for tailwater effects on the breach outflow (i.e., to find whether the water downstream is restricting the free flow through the breach). This comparison and (if necessary) correction allows the model to provide the most accurate prediction of maximum breach outflow which properly accounts for the effects of tailwater depth downstream of the dam.

## 2.2 Channel Description

The river channel downstream of the dam to the specified routing point is approximated as a prismatic channel by defining a single cross section (an average section that incorporates the geometric properties of all intervening sections via a distance weighting technique) and fitting a mathematical function that relates the section's width to depth. This prismatic representation of the channel allows easy calculation of flow area and volume in the downstream channel which is required to accurately predict the amount of peak flow attenuation.

Approximating the channel as a prism requires three steps. First, topwidth vs. depth data must be obtained from topographic maps or survey

notes. For each depth ( $h_i$ ), a distance weighted topwidth  $\bar{B}_i$  is defined producing a table of values that may be used for fitting (using least-squares or a log-log plot) a single equation of the form  $B = Kh^m$  to define the prismatic channel geometry.

For rivers with very steep valley side-walls adjacent to the channel (see Fig. 1a), an additional parameter ( $h_v$ ) may be specified to indicate the depth at which the channel geometry no longer follows the  $B = Kh^m$  relation. As can be seen in Fig. 1b, this feature allows for a more accurate representation of the true channel-valley shape.



Fig. 1a. Typical Downstream Cross-Section

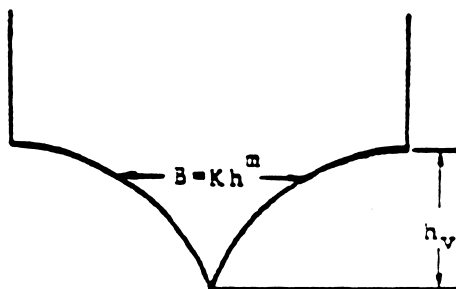


Fig. 1b. Approximated Prismatic Downstream Cross-Section

### 2.3 Downstream Routing

After the maximum breach outflow and stage have been calculated, it is necessary to route the flow downstream. This routing is achieved by employing dimensionless curves developed using the NWS DAMBRK model. These dimensionless curves are grouped into families (see Appendix Ia) and have as their X-coordinate the ratio of the downstream distance (from the dam to a selected cross section) to a distance parameter computed using the equations given in Appendix I. The Y-coordinate of the curves used in predicting peak downstream flows is the ratio of the peak flow at the selected cross section to the computed peak flow at the dam.

The distinguishing characteristic of each curve family is the Froude number developed as the floodwave moves downstream. The distinguishing characteristic of each member of a family is the ratio of the volume in the reservoir to the average flow volume in the downstream channel. Thus it may be seen that to predict the peak flow of the floodwave at a downstream point the user must first determine the desired distinguishing characteristic of the curve family and member. This determination is based on the calculation of the Froude number and the volume ratio parameter. To specify the distance in dimensionless form, the distance parameter must also be computed. The equations required for these computations, as well as the equations for determining the peak stage at forecast points are presented in Appendix I.

The time of occurrence of the peak flow at a selected cross section is determined by adding the time of failure to the peak travel time from the dam to that cross section. The travel time is computed using the kinematic wave velocity which is a known function of the average flow velocity throughout the routing reach. The times of first flooding and "de-flooding" of a particular elevation at the cross section may also be determined when the user provides the selected elevation.

### III. MODEL CONFIGURATIONS

The SMPDBK model was developed for use in dam failure analyses when time is short or where main frame (large) computer facilities are unavailable to the user. For the first reason above, the model's data requirements have been kept to a minimum. For the second reason, the model's computational requirements have also been kept to a minimum such that it can be employed with small (mini- or micro-) computer systems or even with a simple, non-programmable, hand-held calculator. To allow for these three applications, the model has three versions. The first version, known as the "manual" version, allows the user, employing only an inexpensive calculator and the routing curves provided in Appendix Ia, to produce forecasts of the dam-break floodwave peak discharges, stages, and travel times at selected downstream points. To accomplish this, the user simply follows the step-by-step guide provided in Appendix I for making the required calculations, providing data input where necessary, and selects the indicated values from the routing curves.

The second version, known as the "FORTRAN Mini/Micro-computer" version has been, as its name implies, developed for use on mini-computers or micro-computers with a FORTRAN compiler. The computational procedure followed by this version is the same as that followed in the manual version; however, the computations are made by the computer rather than the user, thus significantly reducing the time required to prepare a forecast. This version was developed for interactive use and prompts the user for information (data) as it is needed, automatically using default values (where applicable) when the user is unable to provide them. Included in this version is a "short" option that may be employed when the only channel cross-section data available is that describing the dam face; the model assumes a constant channel configuration and provides forecast information at seven user-specified points downstream. A copy of the FORTRAN code for this version and an example run are provided in Appendix II.

The third version of the model, or the "micro-BASIC" version, has been developed for use on micro-computers with BASIC compilers. This version performs the same computations and requires the same data input as the FORTRAN version and its storage requirements are small enough for it to fit on most micro-computers. The "short" option available in the FORTRAN version is provided in this version as well. Also, a hybrid of this version has been developed for use on a hand-held, programmable computer (See Appendices IIIa and IV). A listing of the BASIC code and an example run is included in Appendix III.

#### IV. CONCLUDING REMARKS

In both real-time forecasting and disaster preparedness planning, there is a clear need for a fast and economical method of predicting dam-break floodwave peak stages and travel times. The SMPDBK model fills this need, producing such predictions quickly, inexpensively and with reasonable accuracy. For example, in test analyses of the Teton and Buffalo Creek dam failures, approximating the channel as a prism, calculating the maximum breach outflow and stage at the dam, defining the routing parameters, and evaluating the peak stage and travel time to the forecast points required less than 20 minutes of time with the aid of a non-programmable hand-held calculator while the average error in forecasted peak flow and travel time was 10-20% with stage errors of approximately 1 ft. Furthermore, comparisons of SMPDBK model results with DAMBRK model results from test runs of theoretical dam breaks show the simplified model produces average errors of 10% or less. The authors had the advantages, however, of prior experience with the model and possession of all required input data, the collection of which consumes precious warning response time in a dam-break emergency.

To help reduce the time required for data collection, default values for some of the input data are presented in Appendix I. These default values may be used by dam-break flood forecasters when time is short and reliable data are unavailable. Additionally, to help further reduce the time needed to employ the model, computerized versions of the model have been developed for FORTRAN-compatible mini-computers and for BASIC language micro-computers and programmable hand-held computers (see Appendices II and III.)

The SMPDBK model is not only useful in a dam-break emergency, it is also suitable for pre-computation of flood peak elevations and travel times prior to a dam failure. Pre-computation of dam failures allows those responsible for community preparedness to delineate danger areas downstream should the dam fail. Ideally, to obtain the most reliable estimate of probable flood elevations and travel times, the sophisticated NWS DAMBRK model would be used in a long-term disaster preparedness study where sufficient computer resources are available. However, for short term studies with limited resources, the SMPDBK model will be most helpful in defining approximate peak stages, discharges, and travel times.

In closing, the authors would like to stress that while the SMPDBK model can be a very useful tool in preparing for and during a dam failure event, the user must keep in mind the model's limitations (Fread 1981). First of all, as with all dam breach flood routing models, the validity of the SMPDBK model's prediction depends upon the accuracy of the required

input data, whether these data be supplied by the user or provided as default-"most probable" values by the model. To produce the most reliable results, the user should endeavor to obtain the best estimates of the various input parameters that time and resources allow. Secondly, because the model assumes normal, steady flow at the peak, the backwater effects created by downstream channel constrictions such as bridge embankments or dams cannot be accounted for and the model will predict peak depths upstream of the constriction that may be substantially lower than those actually encountered, while peak depths downstream of the constriction may be over predicted. Finally, because the "slowing down" of the floodwave caused by temporary off-channel dead storage is not accounted for by the model, the predicted time to peak at a certain point may be somewhat shorter than the actual time to peak. Recognizing these limitations and exercising good engineering judgement, the SMPDBK model user may obtain useful dam break flood inundation information with relatively small expense of time and computing resources.

#### V. BIBLIOGRAPHY

- Davies, W.E., J.F. Bailey, and D.B. Kelly, 1972: "West Virginia's Buffalo Creek Flood: A Study of the Hydrology and Engineering Geology." Geological Survey, Circular 667, U.S. Geological Survey, Washington, D.C., 1972, 32 pp.
- Fread, D.L., 1977: "The Development and Testing of a Dam-Break Flood Forecasting Model." Proceedings, Dam-Break Flood Modelling Workshop, U.S. Water Resources Council, Washington, D.C., 1977, pp. 164-197.
- Fread, D.L., 1980: "DAMBRK: The NWS Dam-Break Flood Forecasting Model." Office of Hydrology, National Weather Service (NWS), Silver Spring, Md., 1980, 37 pp.
- Fread, D.L., 1981: "Some Limitations of Dam Breach Flood Routing Models," Preprint, ASCE Fall Convention, St. Louis Missouri, Oct 26-30, 1981.
- McQuivey, R.S. and T.N. Keefer, 1975: "Application of a Simple Dambreak Routing Model." Proceedings, 16th Congress IAHR, Vol. II, Sao Paulo, Brazil, 1975, pp. 315-324.
- Ray, H.A., L.C. Kjelstrom, Z.G. Crosthwaite, and W.H. Low, 1976: "The Flood in Southeastern Idaho from the Teton Dam Failure of June 5, 1976." Unpublished open file report, U.S. Geological Survey, Boise, Idaho.
- Saakas, J.G., and T. Strelkoff, 1973: "Dam-Break Flood in a Prismatic Dry Channel." Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY12, Proc. Paper 10233, Dec. 1973, pp. 2195-2216.
- SCS, 1979: "Simplified Dam-Breach Flood Routing Procedure." Technical Release No. 66, Engineering Division, USDA Soil Conservation Service, 1979, 23 pp.
- U.S. Army Corps of Engineers, 1975: National Program of Inspection of Dams, Vol. I-V, Dept. of the Army, Office of Chief of Engineers, Washington, D.C.

## **APPENDIX I**

### **THE NWS SIMPLIFIED DAM BREAK MODEL PROGRAM DOCUMENTATION AND USERS' GUIDE WITH EXAMPLE PROBLEM**

The following User Guide is intended to familiarize the user with the Simplified Dam Break Model's data requirements and the computations it performs in producing the information necessary for delineating endangered areas downstream of a failed dam. Briefly stated the model calculates the maximum outflow at the dam, evaluates how this flow will be reduced as it moves from the dam to the downstream point(s) specified by the user, determines the depth attained by this flow at the point(s) and finally computes the time required for the flow to reach the point(s). All of these calculations utilize information (data) supplied by the user, a summary of which is given below. The user should note that while the model supplies default values for many of these input variables, the most accurate results are produced when the most accurate data is entered and the authors strongly urge the use of the best possible estimates of variable values for each study case.

Table A. Simplified Dam Break Model Input Variables

Dam Crest Elevation (Top of Dam)
Final Breach Bottom Elevation
Reservoir Storage Volume
Reservoir Surface Area
Final Breach Width
Time Required for Breach Formation
Turbine / Spillway / Overtopping Flow (at time of failure)
Topwidth (distance between equal contours) vs. (contour)
Elevation data for one or more downstream river valley
cross-sections
Manning Roughness Coefficient (estimate for each given
cross-section)
Flood Depth (depth at which flooding becomes a problem)
at each cross-section
Height of Valley Wall (required for routing through
canyons)
Channel Bottom Slope (required if only one downstream
cross-section given)

## MAXIMUM BREACH OUTFLOW DISCHARGE AND DEPTH CALCULATION

To determine the maximum breach outflow ( $Q_{bmax}$ ) from the dam the user must first obtain the following data values:

- $A_s$  - Reservoir surface area (acres) at maximum pool level
- $H$  - Depth (ft) of maximum pool level above final breach elevation
- $B_r$  - Average final breach width (ft)
- $t_f$  - Time of failure (minutes)
- $Q_0$  - Additional (non-breach) outflow (cfs) at time  $t_f$  (i.e., spillway flow, turbine flow, and/or crest overflow) (optional data value, may be set to 0.)

Defaults: The reservoir surface area may be estimated to be equal to twice the reservoir volume divided by the dam height. The breach depth,  $H$ , may be set equal to the height of the dam. The average final breach width may be estimated to be one to three times the height of the earth dam and the entire width of a concrete arch dam. The time of failure may be estimated to be near zero for a concrete arch dam (see following subsection on instantaneous failure) while the breach erosion rate for earth dams is generally around 2 feet per minute (i.e.,  $t_f$  minutes =  $H/3$ ).

These values are substituted into the broad-crested weir flow equation to yield the maximum breach outflow ( $Q_{bmax}$ ) in cfs, i.e.

$$Q_{bmax} = Q_0 + 3.1 B_r \left( \frac{C}{\frac{t_f}{60} + \frac{C}{\sqrt{H}}} \right)^3 \quad (1)$$

$$\text{where } C = \frac{23.4 A_s}{B_r} \quad (2)$$

### Instantaneous Failure

In some situations where a dam fails very rapidly, the negative wave that forms in the reservoir may significantly affect the outflow from the dam. In such cases, a special equation (Eq. (1a)) must be used to compute the maximum breach outflow. The constraint that indicates whether Eq. (1a) should be used is as follows:

If time of failure in min. ( $t_f$ ) < 0.001 \* Height of dam in feet ( $H_d$ ), then:

$$Q_{bmax} = 3.1 B_r (I_v I_n) H_d^{3/2} \quad (1a)$$

where:

$$I_v = \left[ 1.0 + 0.148 \left( \frac{B_r}{B} \right)^2 (n+1)^2 - 0.083 \left( \frac{B_r}{B} \right)^3 (n+1)^{5/2} \right]^{3/2} \quad (3)$$

$$I_n = \left[ 1.0 - 0.5467 \left( \frac{B_r}{B} \right) (n+1)^{1/2} + 0.2989 \left( \frac{B_r}{B} \right)^2 (n+1)^{1/4} - 0.1634 \left( \frac{B_r}{B} \right)^3 (n+1)^{1/8} + 0.0893 \left( \frac{B_r}{B} \right)^4 (n+1)^{1/16} - 0.0486 \left( \frac{B_r}{B} \right)^5 (n+1)^{1/32} \right]^{3/2} \quad (4)$$

where:  $B_r$  = Breach width  
 $B$  = Valley topwidth at dam crest  
 $H_d$  = Height of dam  
 $n$  = Channel fitting coefficient

### Maximum Depth Calculation

Once the maximum outflow has been calculated, the maximum depth ( $h_{\max}$ ) at the downstream face of the dam must be determined. To do this, the cross section (X-section) at the downstream face of the dam must first be "fitted" (approximated) with an equation of the form  $B = Kh^m$  (topwidth =  $K \times \text{depth}^m$ ) where  $K$  and  $m$  are the fitting coefficients computed using the following least squares algorithm:

$$m = \frac{\sum [(\log h_i) (\log \bar{B}_i)] - \frac{(\sum \log h_i) (\sum \log \bar{B}_i)}{I}}{\sum (\log h_i)^2 - \frac{(\sum \log h_i)^2}{I}} \quad (5)$$

$$\log K = \frac{\sum \log \bar{B}_i}{I} - m \left( \frac{\sum \log h_i}{I} \right) \quad (6)$$

$$K = 10^{(\log K)} \quad (6a)$$

where:  $\bar{B}_i$  is the topwidth associated with depth  $h_i$ ,  $i = 1, 2, 3, \dots, I$  (total number of topwidths)

For rivers with very steep valley side-walls adjacent to the channel (see figure 1a), an additional parameter ( $h_v$ ) may be specified to indicate the depth at which the channel geometry no longer follows the  $B = Kh^m$  relation. As can be seen in Figure 1b, this feature allows for a more accurate representation of the true channel-valley shape.

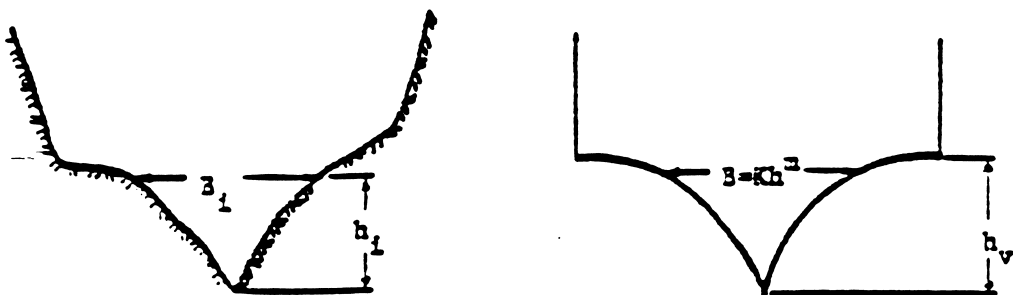


Figure 1 (a) and (b)

Once the  $K$  and  $m$  parameters have been computed for the dam face cross section, the flow ( $Q_v$ ) that produces the depth ( $h_v$ ) (shown above) must be evaluated and compared with  $Q_{bmax}$  to determine whether the maximum depth is above or below  $h_v$ . The flow  $Q_v$  is evaluated using the Manning equation, i.e.,

$$Q_v = \frac{1.49}{n} S^{1/2} \left( \frac{K}{(m+1)^{5/3}} \right) h_v^{(m+5/3)} \quad (7)$$

where:  $n$  = Manning roughness coefficient

$S$  = Channel bottom slope immediately downstream of dam

$K$  &  $m$  = Channel fitting coefficients

Note: The Manning roughness coefficient may be estimated to be 0.04 for a cross-section located in an area of pastureland, .07 for a moderately wooded area and 0.10-0.15 for a heavily wooded area (use the higher value to account for effects caused by significant amounts of debris in the downstream valley).

If  $Q_v$  is found to be greater than  $Q_{bmax}$ , then  $h_v$  must be greater than  $h_{max}$ , and  $h_{max}$  is determined using the relation:

$$h_{max} = \left( \frac{Q_{bmax}}{a} \right)^{1/b} \quad (8)$$

$$\text{where } a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} \quad (9)$$

$$b = m + 5/3 \quad (10)$$

If however,  $Q_v$  is found to be less than  $Q_{bmax}$  then  $h_v$  must be less than  $h_{max}$ , and  $h_{max}$  is calculated as follows:

$$h_{max} = \rho (Q_{bmax})^{3/5} + \gamma h_v \quad (11)$$

where: 
$$\rho = \left[ \frac{1}{a(m+1)^{5/3} h_v^m} \right]^{3/5} \quad (12)$$

$$\gamma = \frac{m}{m+1} \quad (13)$$

and  $a$  is defined by Eq. (9).

#### Submergence Correction<sup>1</sup>

The maximum breach outflow must be corrected iteratively for submergence resulting from tailwater effects if the computed maximum outflow stage ( $h_{max}$ ) is greater than  $(0.67 h_{weir})$  where  $h_{weir}$  is the head over the weir (breach) at time  $t_f$  as expressed by the following relation:

$$h_{weir} = \left( \frac{C}{t_f + \frac{C}{\sqrt{H}}} \right)^2 \quad (14)$$

where  $C$  is defined by Eq. (2).

If the ratio of  $(h_{max}/h_{weir})$  is greater than 0.67, a submergence correction factor must be computed as follows:

$$K_s^* = 1 - 27.8 \left[ \frac{h_{max}}{h_{weir}} - 0.67 \right]^3 \quad (15)$$

---

<sup>1</sup> Note: Submergence correction need not be made in the event of an instantaneous failure.

This value for  $K_s^*$  is substituted into Eq. (15a) to obtain an averaged submergence correction factor given by the following:

$$K_s^k = \frac{K_s^* + K_s^{k-1}}{2} \quad (15a)$$

where the  $k$  superscript is the iteration counter and the first iteration value for  $K_s^0$  is 1.

Note: In some situations, the correction factor  $K_s^*$ , may be computed to be less than 0.8, in which case it should be reset to 0.8 to prevent over-compensating for submergence in the first iteration.

This correction factor is applied to the breach outflow to compute the corrected breach outflow as follows:

$$Q_b^k = K_s^k \cdot Q_b^{k-1} \quad (16)$$

where:  $Q_b^{k-1} = Q_{bmax}$  in the first iteration

The corrected breach outflow ( $Q_b^k$ ) is then compared with  $Q_v$ , and a new outflow depth ( $h_{max}^k$ ) is calculated using Eq. (8) or (11). Also, because there is decreased flow through the breach, there is less drawdown. Thus, the head over the weir ( $h_{weir}$ ) must be recalculated using the relation:

$$h_{weir}^k = h_{weir}^{k-1} + \frac{Q_b^{k-1} - Q_b^k}{2\lambda_s} \frac{C_d}{(sq.ft.)} \quad (17)$$

Now the ratio of the two new values,  $h_{max}^k/h_{weir}^k$  is used in Eq. (15) to compute a new submergence correction factor.

If the new maximum breach outflow computed via Eq. (16) is significantly different ( $\pm 5\%$ ) from that computed in the previous iteration, the procedure is repeated. Generally, within two or three iterations the  $K_s$  value will converge and a suitable value for the maximum breach outflow ( $Q_b$ ) is achieved which properly accounts for the effects of submergence.

## EXAMPLE MAXIMUM BREACH OUTFLOW DISCHARGE AND DEPTH CALCULATION

The following data on the reservoir and breach are gathered:

Reservoir surface area at max pool level,	$A_s = 350$ acres
Height of max pool level above final breach elevation	$H = 50$ ft
Average final breach width (estimated),	$B_T = 100$ ft
Time to maximum breach size (time of failure),	$t_f = 45$ minutes
Additional outflow at time $t_f$ ,	$Q_0 = 5000$ cfs

These values are substituted into Eqs. (1) and (2) to determine the maximum breach outflow, i.e.

$$C = \frac{23.4 (350)}{100} = 81.9$$

$$Q_{bmax} = 3.1(100) \left( \frac{\frac{81.9}{\frac{45}{60}} + \frac{81.9}{\sqrt{50}}}{\sqrt{50}} \right)^3 + 5000 = \underline{\underline{95800 \text{ cfs}}}$$

From a topographic map the following cross-sectional data is obtained:

Table 1 — X-SECTION DATA FOR EXAMPLE PROBLEM

Mile 0.0		Mile 12.3		Mile 40.5	
Elev	Topwidth	Elev	Topwidth	Elev	Topwidth
5532	0	5483	0	5370	0
5540	480	5490	400	5380	590
5550	900	5500	770	5390	930
5560	1300	5510	1330	5400	1440
5570	1350	5520	1400	5410	1460

To compute the  $K$  and  $n$  fitting coefficients for the  $X$ -section at the downstream face of the dam, the  $X$ -sectional data at mile 0.0 is reduced by subtracting the channel invert elevation from the elevation associated with each topwidth, producing table 2.

Table 2 — TOPWIDTH VS. DEPTH AT DAM FOR EXAMPLE PROBLEM

<u>Depth</u>	<u>Topwidth</u>
0	0
8	480
18	900
28	1300
38	1350

Examination of this reduced data indicates that, because the topwidths increase very slowly above a depth of approximately 30 feet, a reasonable value for  $h_v$  is 30.

This reduced data is used in Eqs. (5)-(6a) to determine the  $K$  and  $n$  coefficients of the equation  $B = Kh^m$ .

Table 3 — VALUES USED IN LEAST SQUARES COMPUTATION  
OF  $K$  AND  $n$  COEFFICIENTS AT DAM

1	$h_i$	$\log h_i$	$(\log h_i)^2$	$B_i$	$\log B_i$	$(\log h_i)(\log B_i)$
1	8	0.90	0.82	480	2.68	2.42
2	18	1.26	1.58	900	2.95	3.71
3	28	1.45	2.09	1300	3.11	4.51
-	$h_v$	-	-	-	-	-
4	<u>30</u>	<u>1.48</u>	<u>2.18</u>	1310*	<u>3.12</u>	<u>4.60</u>
-	-	5.08	6.67		11.37	13.24

\*Linearly interpolated.

$$a = \frac{15.24 - \frac{(5.08)(11.87)}{4}}{6.67 - \frac{(5.08)^2}{4}} = 0.78$$

$$\log K = \frac{11.87}{4} - 0.78 \left( \frac{5.08}{4} \right) = 1.98$$

$$K = 10^{\log K} = 95.76$$

To calculate the maximum depth at the dam, the value of  $Q_v$  must first be determined using Eq. (7). The channel bottom slope immediately downstream of the dam,  $S$ , is determined from the X-sectional data, i.e.,

$$S = \frac{(5532 - 5483)}{(12.3)(5280)} = 0.0008$$

and the Manning roughness coefficient,  $n$ , for the downstream channel is estimated to be 0.045.

Thus  $Q_v$  is calculated as follows:

$$Q_v = \frac{1.49}{.045} (.0008)^{1/2} \left( \frac{95.76}{(0.78 + 1)^{5/3}} \right) (30) (0.78 + 5/3) = 136976 \text{ cfs}$$

Because  $Q_v$  is greater than  $Q_{b\max}$  (95800 cfs), Eq. (3) is used to determine  $h_{\max}$ , i.e.,

$$a = \frac{1.49}{.045} (.0008)^{1/2} \left( \frac{95.76}{(0.78 + 1)^{5/3}} \right) = 33.31$$

$$b = 0.78 + 5/3 = 2.45$$

$$h_{\max} = \frac{95800}{33.31} (1/2.45)$$

$$\underline{h_{\max} = 25.81 \text{ ft}}$$

To check for submergence, the head over the weir (breach),  $h_{\text{weir}}$ , is calculated using Eq. (14), i.e.,

$$h_{\text{weir}} = \left( \frac{81.9}{\frac{45}{60} + \frac{81.9}{\sqrt{50}}} \right)^2 = 44.1 \text{ ft}$$

Then the ratio of  $h_{\text{max}}$  to  $h_{\text{weir}}$  is checked as follows:

$$\frac{h_{\text{max}}}{h_{\text{weir}}} = \frac{23.81}{44.10} = 0.59$$

Because the ratio is less than 0.67, the tailwater does not affect breach outflow.

## DOWNSTREAM ROUTING

The peak outflow discharge determined in the preceding step may be routed downstream using the dimensionless routing curves in Appendix Ia. These curves were developed from numerous executions of the NWS DAMBRK Model and they are grouped into families based on the Froude number associated with the floodwave peak. To determine the correct family and member curve that most accurately predicts the attenuation of the flood, the user must first define certain routing parameters.

Prior to defining the routing parameters, however, the user must first describe the river channel downstream from the dam to the first routing point as a prism. To describe the river channel downstream of the dam as a prismatic channel the user must reduce the topwidth vs. elevation data (see table 1) for the first two cross-sections. This cross-section data is reduced (see table 4, p. I-24) to depth (h) vs. topwidth (B) data by subtracting the channel invert elevation from the elevation associated with each topwidth at a given cross-section. From this reduced data, an average cross section may be determined using the following algorithm.

For each depth ( $h_i$ ), the average topwidth ( $\bar{B}_i$ ) is given by the relation:

$$\bar{B}_i = \frac{(B_{i,1} + B_{i,2})}{2} \quad (18)$$

Where:  $h_i$  is the  $i^{\text{th}}$  depth,  $i = 1, 2, 3 \dots I$  (number of topwidths per X-Section)

$B_{i,j}$  is the  $i^{\text{th}}$  topwidth (corresponding to the  $i^{\text{th}}$  depth  $h_i$ ) at the  $j^{\text{th}}$  X-section where  $j = 1, 2$

$\bar{B}_i$  is the average  $i^{\text{th}}$  topwidth

Note: The preceding calculation is unnecessary when depth vs. topwidth data is available for only one X-section.

The table of values produced by defining an average topwidth ( $\bar{B}_i$ ) for each depth ( $h_i$ ) may then be used for fitting (using least squares or a log-log plot) a single equation of the form  $B = Kh^m$  to define the prismatic channel geometry. The fitting coefficients  $K$  and  $m$  may be computed using the following least squares algorithm:

$$m = \frac{\sum [(\log h_i) (\log \bar{B}_i)] - \frac{(\sum \log h_i) (\sum \log \bar{B}_i)}{I}}{\sum (\log h_i)^2 - \frac{(\sum \log h_i)^2}{I}} \quad (5a)$$

$$\log \bar{K} = \frac{\sum \log \bar{B}_i}{I} - m \left( \frac{\sum \log h_i}{I} \right) \quad (6b)$$

$$\bar{K} = 10^{(\log K)} \quad (6c)$$

Using these average cross-section fitting coefficients, the user must recompute the maximum depth at the dam using Eq. (8) or (11). Note, however, that a new check for submergence need not be made. This redefined value for  $h_{\max}$  will be used in computing the routing parameters given in the following section.

#### Routing Parameters

The distance parameter ( $X_c$ ) is calculated using either Eq. (19) or (19a). If the height ( $H_d$ ) of the dam is less than  $h_v$  (defined earlier), the distance parameter is computed as follows:

$$X_c \text{ (ft)} = \frac{(\bar{m}-1)}{\bar{K}} \left( \frac{VOL}{H_d^{\bar{m}-1}} \right) \left( \frac{5}{1 + 4 (0.5)^{\bar{m}-1}} \right) \quad (19)$$

where:  $VOL_r$  = volume in reservoir (cubic ft)  
 $\bar{K}$  &  $\bar{m}$  = average channel geometry fitting coefficients  
 $H_d$  = height of dam (ft)

If the height ( $H_d$ ) of the dam is greater than  $h_v$ , the equation for  $X_c$  is given by the following:

$$X_c \text{ (ft)} = \frac{6 VOL_r}{\bar{K} h_v^{\bar{m}} \left( 3H_d - 5 \frac{\bar{m} h_v}{\bar{m}+1} \right)} \quad (19a)$$

Within the distance ( $X_c$ ) in the downstream reach, the floodwave attenuates such that the depth at point  $X_c$  is  $h_x$  (see figure 2), which is a function of the maximum depth ( $h_{max}$ ). The average depth ( $\bar{h}$ ) in this reach is:

$$\bar{h} = \frac{h_{max} + h_x}{2} = \theta h_{max} \quad (20)$$

where  $\theta$  is an empirical weighting factor that must be determined iteratively. The starting estimate for  $\theta$  is 0.95.

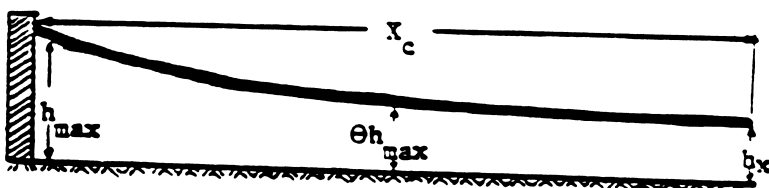


Figure 2

The average hydraulic depth ( $D_c$ ) in the reach is given by Eq. (21) or (22) as follows:

$$\text{CASE I: If } (Q_{b_{\max}} < Q_v), \text{ then } D_c = \frac{\theta h_{\max}}{m+1} \quad (21)$$

$$\text{CASE II: If } (Q_{b_{\max}} > Q_v), \text{ then } D_c = \theta h_{\max} - h_v + \frac{h_v}{m+1} \quad (22)$$

The average velocity in the reach is given by the Manning equation, i.e.:

$$V_c = \frac{1.49}{n} S^{1/2} (D_c)^{2/3} \quad (23)$$

where  $S$  is the slope of the channel from the dam to the routing point.

The average velocity ( $V_c$ ) and hydraulic depth ( $D_c$ ) are substituted into Eq. (24) to determine the average Froude number ( $F_c$ ) in the reach as follows:

$$F_c = \frac{V_c}{\sqrt{g D_c}} \quad (24)$$

where:  $g = 32.2 \text{ ft/sec}^2$  (acceleration of gravity)

The dimensionless volume parameter ( $V^*$ ) that identifies the specific member of the curve family for the computed Froude number is the ratio of the reservoir storage volume to the average flow volume within the  $X_c$  reach. The average cross sectional area of flow ( $A_c$ ) is given by Eq. (25) or (26) as follows:

$$\text{CASE I: If } (Q_{b_{\max}} < Q_v), \text{ then } A_c = \bar{K}(\theta h_{\max})^{\frac{m}{m+1}} D_c \quad (25)$$

$$\text{CASE II: If } (Q_{b_{\max}} > Q_v), \text{ then } A_c = \bar{K}h_v^{\frac{m}{m+1}} D_c \quad (26)$$

The volume parameter ( $V^*$ ) is determined by dividing the average flow volume ( $A_c X_c$ ) into the reservoir storage volume ( $VOL_r$ ), i.e.:

$$V^* = \frac{VOL}{A_c X_c}$$

(27)

To summarize this section on routing parameter calculation and to allow for quick reference, the following table of equations is presented.

Parameter	Case I	Equation	Eq. No.	Case II	Equation	Eq. No.
$x_c$ (ft)	$u_d < h_v$	$x_c = \frac{\bar{m}+1}{\bar{K}} \frac{\text{VOL. r}}{u_d^{\bar{m}+1}} \left[ \frac{6}{1+4(0.5)^{\bar{m}+1}} \right]$	(19)	$u_d > h_v$	$x_c = \left[ \frac{\text{VOL. r}}{\bar{K} h_v^{\bar{m}}} \right] \left[ \frac{6}{3u_d - 5 \frac{h_v}{\bar{m}+1}} \right]$	(19a)
$u$	all cases	$u = 0.95^u$				
$D_c$ (ft)	$q_{b_{\max}} < q_v$	$D_c = \frac{u h_{\max}}{\bar{m}+1}$	(21)	$q_{b_{\max}} > q_v$	$D_c = \theta h_{\max} - h_v + \frac{h_v}{\bar{m}+1}$	(22)
$V_c$ (ft <sup>3</sup> /u)	all cases	$V_c = \frac{1.49}{n} S^{1/2} (D_c)^{2/3}$	(23)			
$F_c$	all cases	$F_c = \frac{V_c}{\sqrt{B D_c}}$	(24)			
$A_c$ (ft <sup>2</sup> )	$q_{b_{\max}} < q_v$	$A_c = \bar{K} (u h_{\max})^{\bar{m}} D_c$	(25)	$q_{b_{\max}} > q_v$	$A_c = \bar{K} h_v^{\bar{m}} D_c$	(26)
$V^a$	all cases	$V^a = \frac{\text{VOL. r}}{A_c x_c}$	(27)			

Note: Further discussion of the value of  $\theta$  follows.

## Routing Curves

Knowing the value of  $F_c$  and  $V^*$ , the user may consult the specific curve (see Appendix Ia) defined for these values (interpolation may be necessary) and check the original estimate of the value of  $\theta$  used in Eq. (21)-(27). The ordinate of the routing curve at  $X^* = 1$  is the ratio of the peak flow ( $Q_p$ ) at  $X_c$  to  $Q_{b_{\max}}$ . Knowing  $Q_p$ , the user may determine the stage ( $h_x$ ) at  $X_c$  using Eq. (8) or (11) with the average channel fitting coefficients. The value of  $\theta$  is checked by rearranging Eq. (20), i.e.:

$$\theta = \frac{h_{\max} + h_x}{2 h_{\max}} \quad (28)$$

If there is a significant difference in the new value of  $\theta$  from the initial estimate of  $\theta$  (e.g.,  $\pm 5\%$ ), Eqs. (21)-(27) should be recalculated and the new value of  $\theta$  rechecked. Generally, within two passes the value for  $\theta$  will converge.

Knowing the proper routing curve for predicting peak flow attenuation, the user may then non-dimensionalize the distance(s) downstream to the forecast point(s) using the relation.

$$X_i^* = \frac{X_i}{X_c} \quad (29)$$

where  $X_i$  is the downstream distance to the  $i^{\text{th}}$  forecast point,  $i = 1, 2, 3, \dots$

To find the peak flow at  $X_i$ , the user consults the proper family of routing curves and finds the ordinate of the specific  $V^*$  curve at  $X_i^*$ . Multiplying the value of this ordinate by  $Q_{b_{\max}}$  produces the peak flow ( $Q_p$ ) at  $X_i$  miles downstream of the dam.

The time of travel for the floodwave to  $X_i$  is computed by first calculating the reference flow velocity at the midpoint between the dam and  $X_i$ . The user must determine, from the routing curve, the peak flow ( $Q_{x/2}$ )

at  $(X_1/2)$  miles downstream of the dam. This flow is multiplied by the factor  $(0.3 + \bar{m}/10)$  and substituted into Eq. (30) or (31) to find the reference depth ( $h_{ref}$ ).

$$\text{If } Q < Q_v, \text{ then } \dots h = \left(\frac{Q}{a}\right)^{1/b} \quad (30)$$

where  $a$  and  $b$  are defined by Eqs. (9 and 10).

$$\text{If } Q > Q_v, \text{ then } \dots h = \rho Q^{3/5} + \gamma h_v \quad (31)$$

where  $\rho$  and  $\gamma$  are defined by Eq. (12) and (13).

The reference hydraulic depth is given by Eq. (32) or Eq. (33) i.e.:

$$\text{CASE I: If } h_{ref} < h_v, \text{ then } D_{x_1} = \frac{h_{ref}}{m+1} \quad (32)$$

$$\text{CASE II: If } h_{ref} > h_v, \text{ then } D_{x_1} = h_{ref} - h_v + \frac{h_v}{m+1} \quad (33)$$

The reference flow velocity is given by the Manning equation, i.e.:

$$V_{x_1} \text{ (ft/s)} = \frac{1.49}{n} S^{1/2} D_{x_1}^{2/3} \quad (34)$$

This value for  $V_{x_1}$  is substituted into the wave celerity equation (Eq. (35)) to find the wave speed.

$$C \text{ (mi/hr)} = 0.682 V_{x_1} \left[ 5/3 - 2/3 \left( \frac{3}{m+1} \right) \right] \quad (35)$$

The time to peak is then given by Eq. (36) as follows:

$$t_{p_i}(\text{hr}) = t_f(\text{hr}) + \frac{X_i(\text{mi})}{C(\text{mi/hr})} \quad (36)$$

where:  $t_f(\text{hr})$  = time of failure for dam (minutes)/60

To compute the peak depth at mile  $X_i$ , the user must fit  $K$  and  $n$  coefficients for that  $X$ -section by substituting the specific depths and topwidths at mile  $X_i$  into Eqs. (5)-(6a). The peak flow at mile  $X_i$  is then compared with  $Q_v$  (Eq. (7)) and Eq. (30) or (31) is used to find the peak depth ( $h_{x_i}$ ) at mile  $X_i$ .

The user may wish to determine the time at which flooding commences and/or the time at which it ceases. To do this, the user must first specify a flow rate,  $Q_f$  that corresponds with flood depth at the  $X$ -section.

$$Q_f = a h_f^b \quad (37)$$

where  $h_f$  = flood depth

and  $a$  and  $b$  are defined by Eqs. (9)-(10) using the  $K$  and  $n$  coefficients fitted for the  $X$ -section at mile  $X_i$ .

This value for  $Q_f$  is substituted into Eq. (38) to determine the time to flooding,  $t_{fld}$ , as follows:

$$t_{fld}(\text{hr}) = t_{p_i}(\text{hr}) - \left( \frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o} \right) t_f(\text{hr}) \quad (38)$$

where  $t_{p_i}$  is the time to peak calculated in Eq. (36)

$t_f$  is the time of failure for the dam, and

$Q_o$  is the nonbreach (spillway/turbine/overtopping) flow.

To determine the time flooding ceases,  $t_d$ , the value of  $Q_f$  is substituted into the following relation:

$$T_d(hr) = \tau_{p_i}(hr) + \left[ \frac{24.2 \text{ VOL}_r}{Q_{p_i} - Q_o} - \tau_f(hr) \right] \left( \frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o} \right) \quad (39)$$

where  $\text{VOL}_r$  is the reservoir storage volume (ac-ft).

To route the peak flow downstream to cross sections 3,4,..., the user must determine the distance weighted average cross section between the dam and the routing point and fit new  $\bar{K}$  and  $\bar{a}$  parameters to this cross section. From the reduced data, a distance weighted average cross section may be determined using the following algorithm.

For each depth ( $h_i$ ), the distance weighted topwidth ( $\bar{B}_i$ ) is given by the relation:

$$\bar{B}_i = \frac{\frac{(B_{i,1} + B_{i,2})}{2} (X_2 - X_1) + \dots + \frac{(B_{i,J-1} + B_{i,J})}{2} (X_J - X_{J-1})}{(X_J - X_1)} \quad (40)$$

Where:  $h_i$  is the  $i^{\text{th}}$  depth,  $i = 1, 2, 3 \dots I$  (number of topwidths per X-section)

$B_{i,j}$  is the  $i^{\text{th}}$  topwidth (corresponding to the  $i^{\text{th}}$  depth  $h_i$ ) at the  $j^{\text{th}}$  X-section where  $j = 1, 2, 3, \dots J$  (number of X-sections)

$\bar{B}_i$  is the weighted  $i^{\text{th}}$  topwidth

$X_j$  is the downstream distance to the  $j^{\text{th}}$  X-section.

The table of values produced by defining a distance weighted topwidth ( $\bar{B}_i$ ) for each depth ( $h_i$ ) may then be used for fitting a single equation of the form  $B = K h^2$  to define the prismatic channel geometry. The fitting coefficients  $\bar{K}$  and  $\bar{a}$  may be computed using the least squares algorithm given in Eqs. (5a)-(5c).

With these weighted average  $\bar{K}$  and  $\bar{n}$  coefficients, the user must then recompute a peak depth at the dam and new routing parameters using Eqs. (20)-(27). The user may then route the flow to cross-section 3,4, ... J by following the procedure given above.

## EXAMPLE SIMPLIFIED DAMBREAK DOWNSTREAM ROUTING

To route the flow from the dam to the cross section at mile 12.3, the user must first define an average cross section for the reach and then fit  $\bar{K}$  and  $\bar{m}$  coefficients. Reducing the data given in Table 1 produces the following table.

Table 4.—TOPWIDTH VS. DEPTH FOR EXAMPLE PROBLEM

Mile 0.0		Mile 12.3	
Depth	Topwidth	Depth	Topwidth*
0	0	0	0
8	480	8	437
18	900	18	826
28	1300	28	1337
38	1350	38	1407

\*Linearly interpolated to agree with depth value at mile zero.

To determine the average X-section, the depth vs. topwidth data is substituted into the relation given in Eq. (18), i.e.,

for  $h_1 = 8$ ,

$$B_1 = \frac{480 + 437}{2} = 458.5$$

for  $h_2 = 18$ ;

$$B_2 = \frac{900 + 826}{2} = 863$$

for  $h_3 = 28$ ,

$$B_3 = \frac{1300 + 1337}{2} = 1318.5$$

The average cross section data is used in Eqs. (5a)-(6c) to determine the average  $\bar{K}$  and  $\bar{m}$  coefficients for the reach.

Table 5.— VAUES USED IN LEAST SQUARES COMPUTATION OF K AND M

i	$h_i$	$\log h_i$	$(\log h_i)^2$	$B_i$	$\log B_i$	$(\log h_i)(\log B_i)$
1	8	0.90	0.82	458	2.66	2.40
2	18	1.26	1.58	863	2.94	3.69
3	28	1.45	2.09	1318.5	3.12	4.52
-	$h_v$	-	-			
4	30	<u>1.48</u>	<u>2.18</u>	1330.5	<u>3.12</u>	<u>4.61</u>
		5.08	6.67		11.84	15.22

$$\bar{m} = \frac{15.22 - \frac{(508)(11.84)}{4}}{6.67 - \frac{(5.08)^2}{4}} = 0.82$$

$$\log \bar{K} = \frac{11.84}{4} - .82 \left( \frac{5.08}{4} \right) = 1.91$$

$$\bar{K} = 10^{\log \bar{K}} = 81.75$$

With these average  $\bar{K}$  and  $\bar{m}$  coefficients, the max depth at the dam may be recomputed using Eq. (8) as follows:

$$a = \frac{1.49}{0.45} (.0008)^{1/2} \left( \frac{81.75}{(0.82 + 1)^{5/3}} \right) = 27.29$$

$$b = 0.82 + 5/3 = 2.49$$

$$h_{\max} = \frac{95800}{27.29}^{(1/2.49)} = 26.49$$

The data required for the flow routing computations are as follows:

$VOL_r$	= 8750 ac-ft	(reservoir volume)
$H_d$	= 50 ft	(height of dam)
$Q_{b_{max}}$	= 95800 cfs	(max breach outflow)
$h_{max}$	= 26.49 ft	(recomputed max stage at dam)
$h_v$	= 30 ft	(height of valley wall)
$\bar{m}$	= 0.82	(fitting coefficient for average cross section)
$\bar{K}$	= 81.75	(fitting coefficient for average cross section)
$S$	= 0.0008	(channel bottom slope from mile 0.0 to mile 12.3)
$n$	= 0.045	(Manning roughness coefficient)
$h_c$	= 10 ft	(flood stage at mile 12.3)
$Q_0$	= 5000 cfs	(initial flow)

Because ( $H_d > h_v$ ), Eq. (19) is used to compute  $X_c$ .

$$X_c (ft) = \frac{6 VOL_r}{\bar{K} h_v^{\bar{m}} \left( 3H_d - 5 \frac{\bar{m}h_v}{\bar{m}+1} \right)}$$

$$= \frac{6 (8750) \times 43560 (ft^3/ac-ft)}{81.75(30)^{0.82} \left( 3(50) - 5 \frac{.82(30)}{1.32} \right)} = 20600 ft (3.9 mi)$$

The first estimate for  $\theta$  is 0.95.

Because ( $Q_{b_{max}} < Q_v$ ) the hydraulic depth ( $D_c$ ) is given by Eq. (21),  
i.e.:

$$D_c = \frac{\theta h_{max}}{\bar{m} + 1} = \frac{0.95 (26.49)}{1.32} = 13.83$$

The average particle velocity in the  $X_c$  reach is as follows (Eq. (24)):

$$v_c = \frac{1.49}{n} s^{1/2} (D_c)^{2/3} = \frac{1.49}{0.045} (0.0008)^{1/2} (13.83)^{2/3} = 5.24 \text{ ft/s}$$

The Froude number developed by the floodwave is determined by the following equation (Eq. (24)):

$$F_c = \frac{v_c}{\sqrt{g D_c}} = \frac{5.24}{\sqrt{32.2 (13.83)}} = 0.25$$

Because  $(Q_{b_{\max}} < Q_v)$ , the average cross-sectional area ( $A_c$ ) of the wave is given by Eq. (25), i.e.:

$$A_c = K(\theta h_m)^m D_c = 81.75 (0.95(26.49))^{0.82} (13.83) = 16158 \text{ (ft}^2\text{)}$$

The volume parameter ( $V^*$ ) is as follows (Eq. (27)):

$$V^* = \frac{VOL_r}{A_c X_c} = \frac{8750 (43560)}{(16158) (20600)} = 1.15$$

To check the original estimate for  $\theta$ , the family of curves for  $F_c = 0.25$  is examined to find by interpolation the ratio of  $Q_p/Q_{\max}$  for  $(V^* = 1.15)$  at  $(X^* = 1)$ . This ratio is found to be approximately 0.60, indicating that the peak flow at  $X_c$  is

$$Q_p \text{ at } X_c = (Q_p/Q_{b_{\max}}) (Q_{b_{\max}}) = 0.60 (95800) = 57480 \text{ cfs}$$

Because this peak flow is less than  $Q_v$  (Eq. (7)), the depth reached by the floodwave at  $X_c$  is given by Eq. (8) as follows:

$$h_{x_c} = \left( \frac{Q_p}{a} \right)^{1/b}$$

$$a = \frac{1.49}{n} S^{1/2} \left( \frac{\bar{K}}{(\bar{n} + 1)^{5/3}} \right) = 27.29$$

$$b = \bar{n} + 5/3 = 2.49$$

$$h_{x_c} = \left( \frac{57480}{27.29} \right)^{1/2.49}$$

$$h_{x_c} = 21.61$$

This value of  $h_{x_c}$  is substituted into Eq. (23) to re-evaluate the original estimate for  $\theta$  :

$$\theta = \frac{\frac{h_{\max} + h_{x_c}}{2 h_{\max}}}{2} = \frac{26.49 + 21.61}{2 (26.49)} = 0.91$$

Because the re-evaluated  $\theta$  is within ( $\pm 5\%$ ) of the original estimate for  $\theta$ , the initial calculations are acceptable and the attenuation of the flood peak may be predicted using the curves for  $\bar{F}_c = 0.25$  and interpolating  $V^* = 1.15$ .

The dimensionless distances ( $X_1^*$ ) to at mile 12.3 is as follows:

$$X_1^* = \frac{12.3}{x_c} = \frac{12.3}{\left( \frac{20600}{5230} \right)} = 3.15$$

The ratio of  $(Q_p/Q_{b_{\max}})$  at  $X^* = 3.15$  is found by interpolation to be 0.34, indicating the peak flow at mile 12.3 to be  $(0.34)(95800) = 32572$  cfs.

To determine the time to peak at the forecast point, the velocity of flow at the midpoint between the dam and the forecast point must be determined. The ratio of the average peak flow  $(Q_{X/2})$  to  $Q_{b_{\max}}$  in the 12.3 mile reach is found on the curve at  $X^* = 1.58$ .

$$X_1^*/2 = X_1/2X_c = 12.3/2(3.9) = 1.58$$

This ratio is approximately 0.5, indicating the midpoint peak flow to be  $(0.5 \times 95800) = 47900$  cfs. This flow is multiplied by the factor  $(0.3 + \bar{m}/10)$  to compute the reference flow  $Q_{\text{ref}} = (.3 + .82/10) 47900 = 18297$ .

Because this flow is less than  $Q_v$ , Eq. (30) is used to determine the reference peak depth in the reach as follows:

$$h_{\text{ref}} = \left( \frac{Q_{\text{ref}}}{a} \right)^{1/b} = \left( \frac{18297}{27.29} \right)^{1/2.49} = 13.65 \text{ ft}$$

The midpoint reference hydraulic depth in the reach is found using Eq. (32), i.e.:

$$D_{\text{ref}} = \frac{h_{\text{ref}}}{m+1} = \frac{13.65}{1.82} = 7.50$$

The average particle velocity is determined by substituting this value of  $D_{x_1}$  into Eq. (34) as follows:

$$v_{x_1} = \frac{1.49}{n} S^{1/2} (D_{x_1})^{2/3} = \frac{1.49}{0.045} (.0008)^{1/2} (7.50)^{2/3}$$

$$v_{x_1} = 3.43 \text{ ft/sec}$$

Thus, the wave celerity as determined with Eq. (35) is:

$$C = 0.682 (3.48) \left[ 5/3 - 2/3 \left( \frac{.82}{1.82} \right) \right] = 3.25 \text{ mi/hr}$$

The time of travel for the flood peak to the forecast point at mile 12.3 is given by Eq. (36) as follows:

$$t_p (X=12.3) = t_f + \frac{12.3}{C} = 0.75 + \frac{12.3}{3.25}$$

$$t_p = 4.54 \text{ hrs}$$

To compute the peak stage at mile 12.3, the user must first compute the  $K$  and  $a$  coefficients for that cross section. This is done by first reducing the X-sectional data at mile 12.3 from table 1 as follows:

Table 6 — TOPWIDTH VS. DEPTH DATA AT MILE 12.3

<u>Depth</u>	<u>Topwidth</u>
7	400
17	770
27	1330
37	1400

Substituting this reduced data into Eqs. (5)-(6a) produces a  $K$  value of 71.96 and an  $a$  value of 0.87. These values are substituted with the peak flow into Eq. (8) to compute the peak depth at mile 12.3 as follows:

$$a = \frac{1.49}{.045} (.008)^{1/2} \frac{71.96}{(0.87 + 1)^{5/3}} = 23.12$$

$$b = 0.87 + 5/3 = 2.53$$

$$h_{12.3} = \left( \frac{32572}{23.12} \right)^{1/2.53} = 17.49$$

To determine the time to flooding (floodwave reaching flood stage), the flow,  $Q_f$ , associated with the flood stage,  $h_f$ , (10 ft) must be computed.

$$Q_f = a(h_f)^b = 23.12 (10)^{2.53}$$

$$Q_f = 7834 \text{ cfs}$$

This value for  $Q_f$  is substituted into Eq. (38) to calculate time to flooding,  $t_{fld}$ , as follows

$$\begin{aligned} t_{fld} &= t_p - \left( \frac{Q_p - Q_f}{Q_p - Q_o} \right) t_f \\ &= 4.54 - \left( \frac{32572 - 7834}{32572 - 5000} \right) (0.75) \end{aligned}$$

$$t_{fld} = 3.87 \text{ hours}$$

The time at which the flooding ceases (floodwave drops below flood stage is calculated using Eq. (39), i.e.,

$$\begin{aligned} t_d &= t_p + \left[ \frac{24.2 \text{ VOL}_R}{Q_p - Q_o} - t_f \right] \left( \frac{Q_p - Q_f}{Q_p - Q_o} \right) \\ &= 3.54 + \left[ \frac{24.2 (8750)}{(32572 - 5000)} - 0.75 \right] \left( \frac{32572 - 7834}{(32572 - 5000)} \right) \end{aligned}$$

$$t_d = 10.76 \text{ hours}$$

Routing the flow to mile 40.5 requires the fitting of  $\bar{K}$  and  $\bar{m}$  parameters to the distance weighted average cross section for the reach. The reduced cross-sectional data for the reach is given in table 6.

Table 6 — TOPWIDTH VS. DEPTH FOR EXAMPLE CHANNEL DESCRIPTION

Mile 0.0		Mile 12.3		Mile 40.5	
Depth	Topwidth	Depth	Topwidth*	Depth	Topwidth*
0	0	0	0	0	0
8	480	8	437	8	472
18	900	18	826	18	862
28	1300	28	1337	28	1338
38	1350	38	1407	38	1456

\*Linearly interpolated to agree with depth value at mile zero.

To determine the distance weighted average X-section, the depth vs. topwidth data is substituted into the relation given in (Eq. (40)), i.e., for  $h_1 = 8$ ,

$$\bar{S}_1 = \frac{\frac{480 + 437}{2} (12.3 - 0.0) + \frac{437 + 472}{2} (40.5 - 12.3)}{(40.5 - 0.0)} = 455.7$$

for  $h_2 = 18$ ;

$$\bar{S}_2 = \frac{\frac{900 + 826}{2} (12.3 - 0.0) + \frac{826 + 862}{2} (40.5 - 12.3)}{(40.5 - 0.0)} = 849.8$$

for  $h_3 = 28$ ,

$$\bar{S}_3 = \frac{\frac{1300 + 1337}{2} (12.3 - 0.0) + \frac{1337 + 1338}{2} (40.5 - 12.3)}{(40.5 - 0.0)} = 1331.7$$

Examination of the X-sectional data indicates that, because the topwidths increase very slowly above a depth of approximately 30 ft, a reasonable value for  $h_v$  is 30.

The distance weighted average cross-section data is used in Eqs. (3a)-(6c) to determine the  $K$  and  $m$  parameters of the equation  $B = Kh^m$ .

Table 7 — VALUES USED IN LEAST SQUARES COMPUTATION OF  $\bar{K}$  and  $\bar{m}$  PARAMETERS

i	$h_i$	$\log h_i$	$(\log h_i)^2$	$B_i$	$\log B_i$	$(\log h_i)(\log B_i)$
1	8	0.90	0.82	455.7	2.66	2.39
2	18	1.26	1.58	849.8	2.93	3.67
3	28	1.45	2.09	1331	3.12	4.51
-	$h_v$	-	-	-	-	-
4	30	<u>1.48</u>	<u>2.18</u>	1383	<u>3.14</u>	<u>4.64</u>
		5.08	6.67		11.85	15.23

$$\bar{m} = \frac{15.23 - \frac{(5.08)(11.85)}{4}}{6.67 - \frac{(5.08)^2}{4}} = 0.84$$

$$\log \bar{K} = \frac{11.84}{4} - 0.84 \left( \frac{5.08}{4} \right) = 1.89$$

$$\bar{K} = 10^{\log \bar{K}} = 78.2$$

With these distance weighted average  $\bar{K}$  and  $\bar{m}$  parameters, the new peak depth at the dam is found to be 26.53 feet. This value is used in Eqs. (19)-(27) to compute the following routing parameters.

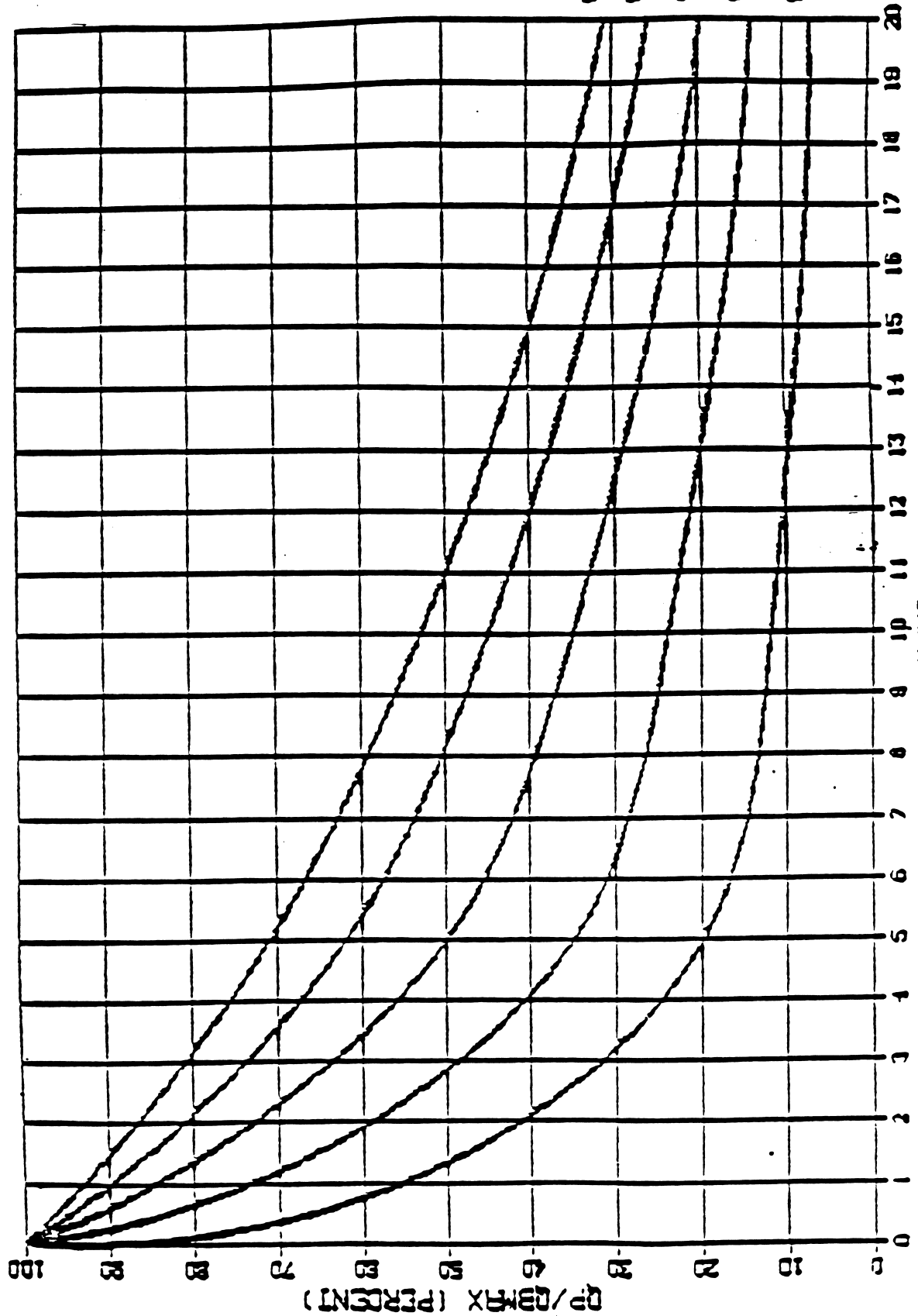
$$\begin{aligned}
X_c &= 20605.6 = 3.9 \text{ mi} \\
\bar{\theta} h_{\max} &= 25.20 \\
D_c &= 13.70 \\
V_c &= 5.22 \\
F_c &= 0.25 \\
A_c &= 16110 \\
V^* &= 1.15
\end{aligned}$$

The dimensionless distance to mile 40.5 is  $(40.5/3.9) = 10.4$ . The ordinate of the  $Q_p/Q_{b\max}$  curve at  $V^* = 1.15$  and  $X^* = 10.4$  is interpolated to be approximately 0.15 indicating the peak flow at mile 40.5 is  $95800 \times 0.15 = 14400$  cfs. The midpoint flow (at mile 40.5/2) is found to be 21737.5. The reference flow is  $(.3 + 0.1 \times 0.84) \times 21737.4 = 8347.16$  which produces a reference depth of 10.05. This value is used Eqs. (32)-(36) to find a time to peak at mile 40.5 of 16.21 hours.

The true  $K$  and  $n$  coefficients at mile 40.5 are 92.1 and 0.80, respectively. These values and the peak flow value are substituted into Eq. (30) to compute a peak depth at mile 40.5 of 11.98 feet. Finally, the time of flooding is found to be 15.85 hours and the time to "deflooding" is found to be 28.48 hours.

**APPENDIX I (a)**  
**THE NWS SIMPLIFIED DAM BREAK MODEL**  
**FLOW ROUTING CURVES**

FC-0.25



FC=0.50

